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THE PRESENT STATUS OF RESEARCH ON SEDIMENT TRANSPORT

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THE PRESENT STATUS OF RESEARCH ON SEDIMENT TRANSPORT

Ning Chien,[1] J.M. ASCE

SYNOPSIS

The problem of sediment transport, as it is known today, is discussed briefly in this paper. Emphasis is placed on the significance and implications of the various phases of the problem. Controversial issues are discussed, and efforts are made to unify some of the highly diversified results.

INTRODUCTION

The research on the movement of sediment along the bed of alluvial rivers was started by DuBoys,(1) who, in 1879, published a formula on bed-load transport rate in which he first introduced the idea of tractive force. It was Gilbert (2) who pioneered in conducting sediment transportation experiments in flumes in 1909-1914. His techniques in making systematic observations and measurements on sediment movement under controlled conditions greatly advanced research in this field, and are still being used today. In the past few decades, because of the rapid expansion of activity in both engineering and geology, the importance of this problem has been increasingly felt and the subjects of interest have been rapidly widened. In consequence, a great number of papers have been written, many of them are not readily accessible. Facing such a huge number of papers and often contradicting statements, the ordinary reader, who is interested in, but has not been able to devote much time to this subject, loses perspective on the development, present status, and possible future directions of sediment research. It is for these readers that this paper was prepared. New information and interpretations are included which may also interest engineers in this field.

In spite of the broadening of the horizons, the basic question in the field of sedimentation remains the same. That is: How much sediment will be carried through a channel by a certain flow in a certain reach of the stream? To answer this question, one must first know how to describe the flow of an alluvial channel, and then proceed to determine how the flow affects the sediment which composes the bed. This paper attempts to outline briefly what is now known, or, more correctly, what we think we know on these and other related phases of the sediment problem.

Hydraulics of an Alluvial Channel Flow

In any open channel flow, there is a definite relationship between the slope, depth, velocity, and the roughness of the boundary. This relationship cannot

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be established easily when the channel is alluvial. An alluvial channel is a channel which contains a bed of loose sediment of the same type that is moved along the bed. Such a channel seldom remains flat and even. Bars and ripples develop at the bed surface at low stages. They become longer when the discharge increases, and eventually may disappear at high flows. At unusually high flows, large and nearly symmetrical sand bars may appear again, accompanied by surface waves in phase with the bottom undulations. These surface waves are commonly termed sand waves, the existence of which is a highly unstable affair. They may appear for a short duration and then disappear again. The sand bars and ripples represent another type of roughness, in addition to the roughness of the grains which compose the channel bed. The problem is complicated because the roughness not only defines the flow, but the flow itself also molds the roughness.

To understand the mechanism of these two different systems of roughness, one must first attempt to identify their respective effects on the flow from the over-all flow conditions. This is necessary because one system of roughness depends largely on the flow, and the other not, at least for the case when the grain resistance behaves hydraulically rough. It becomes even more important when one considers the different roles they play on the sediment motion.

The frictional resistances of these two different systems of roughness are developed in two distinctly different ways. The resistance caused by the grain roughness is the same as the skin friction imposed on the flow by a wall. One recalls that Nikuradse proposed to measure the roughness of a pipe by an equivalent sand size. In an alluvial channel, this equivalent sand size of the boundary becomes the grain size of the bed material itself when the material is uniform, and is equal to a representative size, K_S, when the bed material is a mixture. The transmission of shear to the boundary through skin friction is accompanied by a transformation of flow energy into energy of turbulence. This turbulence is generated in the immediate vicinity of the grains, and has a great effect on the bed-load motion. On the other hand, with sand bars developed at the bed surface, the stream lines of the flow do not follow the bottom contours, but rather separate from the bed surface at the crests of the sand bars. This results in a pressure difference between the front and rear sides of each bar so that part of the flow resistance is transmitted to the boundary by the shape resistance. This part of the flow energy is transformed into turbulence at the interface between the wake and the free stream flow. This turbulence is generated at a considerable distance away from the grains and may not contribute to the bed load motion of the particle.(3)

Most natural alluvial streams are shallow and wide without any appreciable bank friction. For small streams and especially for the distorted stream models, bank friction becomes quite important. Naturally, the turbulence generated at the banks due to the bank friction has nothing to do with the motion of sediment along the bed, and its effect must also be separated and excluded.

Knowing the necessity of separating the individual effects of different friction devices from their over-all effect on the flow, one turns to the question as by what means can this be accomplished and how can the effect of each individual roughness system be described.

Linearity of Friction in Open Channels(4)(5)

It has been found that in flow systems in which a rough surface is arranged in an irregular manner, such that the irregularities are at least five to ten

times larger than the roughness, i.e., where two or more systems of roughness of different order of magnitude are superimposed, the contributions of the individual systems can be calculated independently and added to determine the resulting total frictional force. The total shear stress along the boundary, T, thus can be expressed as

$$\tau / \rho_f = \Sigma \tau_m / \rho_f = \Sigma S_m R_m g \qquad (1)$$

where ρ_f is the density of the fluid, S the energy slope, R the hydraulic radius, g the gravitational acceleration, and the subscript m refers to the "roughness system m." Both R and S may vary with m, and the physical understanding of the process of superimposition can be simplified considerably if one of these two variables is assumed to be common to the different systems of roughness. If R is interpreted as a purely geometric description of the cross section, S must be divided between the different systems of roughness. Thus Cook[6] divided the energy slope of an alluvial channel flow into six component parts. The same scheme was also adopted by Meyer-Peter and Muller.[7] On the other hand, the energy slope is the number of foot pounds which each pound of fluid loses per foot of travel. With S different for different systems, it is difficult to visualize how every pound of fluid contributes energy to all these different systems. A better interpretation is to hold S constant and assume the total energy used at any point in the flow contributes at its full rate to one friction system only. The cross section may be divided into parts contributing to the different systems. If A_m designates the part of the cross-sectional area A contributing energy to the m-th system then $R_m = A_m/p$ is the corresponding hydraulic radius, where p is the common perimeter. Accordingly, one may make the following divisions for an alluvial channel flow:

- Total hydraulic radius R
  - Hydraulic radius of the banks R_w
  - Hydraulic radius of the bed R_b
    - Hydraulic radius of the grain resistance R_b'
    - Hydraulic radius of the bar resistance R_b''

This system will be used throughout the discussion of this section.

The Grain Resistance

The frictional resistance of sediment grains at the bed surface can best be described by the logarithmic formula based on von Karman's similarity theorem. The average velocity, V, of the flow in the vertical may be given by the equation

$$V/(R_b' S g)^{1/2} = \frac{2.3}{\kappa} \log_{10}(12.27 \times R_b'/K_s) \qquad (2)$$

where

x = a corrective parameter for transition smooth-rough, itself, a function of K_s/δ, where δ is the thickness of laminar sublayer. x is unity for a hydraulically rough bed.

k = Karman's universal constant of turbulent exchange.
K_S = representative grain size of the bed material.

Einstein and El-Samni[8] found that the representative grain size of a mixture is equal to D_{65}, the size of which 65% by weight is finer. Meyer-Peter and Muller suggested the value D_{90} instead. There is not enough experimental evidence to indicate which proposal is more reliable. It also involves the question of which part of the bed is taken as the representative bed mixture. Both authors emphasize the coarser part of the mixture which is more important in influencing the roughness.

It has been found from both flume studies and river measurements that the k for a sediment-laden flow is, in general, less than 0.4, the value first determined by Nikuradse from pipe flow with clear water. Vanoni[9][10] and Ismail[11] have suggested that sediment damps the turbulence and reduces the momentum transfer. This results in effect in a reduction of k, and it has been shown that there is a definite tendency of k to decrease with increased sediment load. On the other hand, Laursen and Lin[12] challenged this view, and attributed the change of k to the change of bed roughness.

A change of k means a change of velocity distribution. It will be shown later that theory of suspension gives only the concentration distribution of the suspended sediment, and the rate of suspended-load transport is obtained by integrating the product of local velocity and concentration from the surface of the bed-layer up to the water surface. If velocity distribution changes, one should know what causes the change and how the change can be predicted. The controversial issue over the possible reason behind the change of k must be settled, if further progress a general solution of the sediment transportation problem is to be made.

The k was first introduced by von Karman[13] in his similarity theorem of turbulent flow pattern, from which he established the velocity deficiency law. In circular pipes with radius r, the equation of velocity deficiency assumes the form

$$(V_{max} - V_y)/V_* = -\frac{1}{\kappa}\left[\log_e\left(1-\sqrt{1-y/r}\right)+\sqrt{1-y/r}\right] \qquad (3)$$

where V_{max} is the maximum velocity, V_y the velocity at a distance y from the wall, and V_* the shear velocity at the wall. The law states that at high Reynolds numbers, the ratio (velocity deficiency/shear velocity) is a universal function of the relative distance from the center, and is independent of the Reynolds number, the roughness of the wall, and the shape of the section. From eq. (3) one may conclude that k is independent of the roughness, and the effect of roughness seems to be confined exclusively to the region immediately adjacent to the wall. This has been verified in the laboratory[14][15] by measurements of open channel flows with various cross-sections and roughness; and in the field[16] by analysis of the velocity distribution in non-alluvial stream channels which involve no sediment motion. Von Karman's theory may possibly break down only when the height of the roughness element becomes comparable with the depth of flow, and this seems to be the case according to the findings of Iowa Institute of Hydraulic Research.

The Iowa results, which indicate that k changes with roughness, have not been published as yet. From the statement of Rand,[17] the factor k appears to vary with the size and geometry of the roughness, and the relative roughness h/d, where h is the height of the roughness element and d is the average depth of flow. For instance, k for 1/4-in. square-strip roughness spaced at

1-in. intervals on the bottom of the channel was 0.3 for h/d = 0.3 and 0.37 for h/d = 0.003. However, for the roughness formed by ordinary fly-screen k proved to be 0.37 to 0.40 even for the h/d values up to 0.06.

There exist some other measurements in which the types of roughness element used are similar to those employed in Iowa. Bazin, as reported by Keulegan,[14] used 1 cm. high 2.7 cm. wide wooden strips placed 1 cm. or 5 cm. apart across the rectangular channel and perpendicular to the flow. The ratio of the height of the strips to the hydraulic radius varies from 0.03 to 0.18 for closely spaced strips, and from 0.025 to 0.14 for widely spaced strips. Schlichting[15] studied the effect of roughness on the velocity distribution in a closed rectangular channel, with three sides smooth and the top plate rough. His roughness elements consisted of spheres, spherical segments, cones, short angles, and long angles. The form and arrangement of the short angles are very similar to those of the strips used in Iowa. The ratio of the height of the short angles to the distance from the rough wall to the point near the center of the section, beyond which the flow is affected by the opposite smooth wall, varies from 0.10 to 0.12. The results of these two studies all show that k is 0.4. These two measurements were conducted with considerable care, and the Bazin's results are especially reliable as he used a flume 6-1/2 ft. wide and over 1000 ft. long. In 1951-1953, the Corps of Engineers made a series of measurements on velocity and sediment concentration distribution in the Missouri River near Omaha. A topographic survey of the bottom configuration was made with sufficient accuracy to detect the existence of any sand bars with heights over 0.15 of the depth of flow. At the center flat area of the stream where the topographic survey revealed no such appreciable bottom undulations, the results of velocity distribution measurements indicated that k, as an average, had a value in the neighborhood of 0.25.

The question whether the sediment motion has any effect on the change of k can be settled in a more direct manner. The writer has conducted three sets of experiments with three different sands (1.55, 0.94 and 0.27 mm.). For each experiment, the bottom of the flume was coated by a layer of sand. Velocity distribution with clear water indicated that k is close to 0.4. Enough sand, with the same diameter as the glued on the bed, was then added to the flow to keep the rate of transport as high as possible, and in the mean time, to avoid any significant deposition on the bed. The particles moving along the bed occasionally settled to the bed, but the duration of rest was small because of the high transport rate. Since the bottom was coated by the same sand as that in motion, the widely-scattered, temporary deposition of individual particles at the bed surface in no way affected the bed roughness. Yet the experimental results showed that the value of k was in the neighborhood of 0.30 for all the three experiments.

From the evidences gathered above, one may safely conclude that (a) the sediment motion affects the velocity distribution, and (b) if roughness changes k, the roughness element must be so pronounced that its height becomes comparable with the depth of flow.

Indeed there are reasons to believe that the sediment motion must affect the velocity distribution. In a sediment-laden flow, the distribution of the large sediment grains in the vertical is highly skewed. Near the bed there is a zone where the sediment is heavily concentrated, and the concentration of sediment reduces progressively towards the water surface. For simplicity of argument, the sediment-laden flow may be considered as consisting of two parts: a relatively shallow zone near the bed where the sediment is highly concentrated (heavy fluid zone), and the remaining area where the concentration

is so small that the density of the fluid materially remains unchanged (light fluid zone). The presence of the heavy fluid zone near the bed is very important in the following respects: when the turbulence generated at the bed in the heavy fluid zone enters the light fluid zone, the momentum exchange may become less effective due to the reduction of mass of the exchange flow. In the heavy fluid zone part of the shear may also be transmitted to the boundary through the sediment particle in motion. These two effects, in combination with the decrease in turbulence level because of the energy spent in keeping sediment in suspension, are all accounted for by a reduced value of k.

The effect on the velocity distribution of a heavy sediment concentration close to the bed is demonstrated by the writer's experiment on coarse sediment. The material in motion was confined to a zone close to the bed of about one-tenth of the depth of flow in thickness. Outside of this zone, the water was practically clear. And the result shows a significant reduction in the value of k. There are some indications that the mechanism which causes a change of velocity distribution probably takes place in a zone close to the bed, although the end-product of this effect manifests itself in the main part of the flow where measurements usually are made. The sediment particles suspended in the main part of the flow are of smaller sizes, and it does not take much energy to keep them in suspension. This may account for the apparent constancy in the value of k with depth.

Einstein and the writer[18] proposed to measure the reduction of the value of k by the amount of turbulent energy spent in supporting the sediment in suspension. The rate at which frictional energy is spent per unit weight of fluid and per unit time in supporting the sediment in suspension, $\Sigma \frac{C_s V_s}{VS} \frac{\rho_s - \rho_f}{\rho_s}$, is correlated with k in Fig. 1. Here C_s is the average concentration by weight of a given grain size with settling velocity V_s in the vertical, ρ_s is the density of the solid, and the summation sign designates a summation over all the particles in suspension. The data plotted in Fig. 1 includes both flume studies by Vanoni,[9] Ismail,[11] Kalinske and Hsia,[19] and Chien, and field measurements on the Missouri River at Omaha, Nebraska, and on the Atchafalaya River at Simmsport, La. The scatter of the points is large, as the effect of heavy sediment concentration near the bed may be different under different conditions. Yet the over-all trend that k reduces with increasing parameter $\Sigma \frac{C_s V_s}{VS} \frac{\rho_s - \rho_f}{\rho_s}$ is beyond doubt.

A reduction of k from 0.4 to, say, 0.2 does not mean that the average velocity of the sediment-laden flow will be twice the corresponding sediment-free flow. Evidently eq. (2) is no longer adequate in describing a sediment-laden flow. The development of a new equation on velocity distribution must be waited until the effect of heavy sediment concentration near the bed becomes better understood. It is believed, however, that the difference in average velocity between the sediment-laden flow and the sediment-free flow will not be appreciable.

Bar Resistance

The mechanism of ripple formation remains today a puzzling problem. Bagnold[20] was able to show that the ripple formation on desert sand is closely associated with the motion of saltation of the bed-load particles. While saltation is the dominating mode of transportation of eolian sediment, Kalinske[21] has shown that it is insignificant for sediment motion in water. Exner[22] has published a formula indicating that the rate of change of bed

elevation is proportional to the longitudinal variation of the bottom velocity. Giving a smooth bed to begin with, then, what causes the longitudinal variation of bottom velocity? Anderson[23] suggested that this variation is associated with the surface waves. Observations made at the University of California on the movement of light-weight materials revealed that the bottom undulations can be as high as 1/4 to 1/3 of the water depth, yet the water surface remains almost mirror smooth. Perhaps Anderson's description applies only to the case of sand waves, the last developmental stage of the bottom undulations.

The initiation of ripples has been shown by Liu[24] in Fig. 2, using V_*/V_S and V_*D/V as the characteristic parameters, where D is the grain size. In the same graph the results of Shields[25] on the beginning of bed-load movement and the data of Quraishy[26] on the commencement of general movement of bed-load are also included. The general movement of bed-load is defined by Quraishy as the stage of movement when all the sediment sizes are in motion and the movement is strong enough to develop bed configurations. According to Fig. 2, ripples generally appear and develop right after the bed-load movement begins, but before the movement is strong enough that there may develop a layer close to the bed in which the sediment is heavily concentrated. The same phenomenon was also observed in India.[27] This seems to rule out the possibility that ripple formation is a phenomenon associated with the instability at the interface of two flows with different density.

The ripples and bars change considerably and consistently with the rate of sediment motion along the bed. The sediment motion, as it will be seen later, is related to a flow function of the type

$$\psi' = \frac{\rho_s - \rho_f}{\rho_f} \frac{D_{35}}{R_b' S} \tag{4}$$

where D_{35} is the sediment size of which 35% by weight is finer. Tsubaki, Kawasumi, and Yasutomi[28] have shown that the flow intensity ψ' does govern the dimensions of the sand bars, according to the result of field measurements in irrigation flumes. Similar to eq. (2), the sand bar resistance can be expressed as

$$V/\sqrt{R_b'' S g} = f_1(\psi') \tag{5}$$

It was found that such a universal relationship between $V/\sqrt{R_b'' S g}$ and ψ' exists for natural stream channels.[29] This function may be different for more confined canal flow, and other factors, such as the width of the channel, might be of importance.

Bank Friction

Stream banks usually consist of material different from movable bed material, or are covered with vegetation. Although bank friction can be calculated by a formula similar to eq. (2), it is customary to assume a Manning's n for the banks and compute the friction by the Manning's formula,

$$V = \frac{1.486}{n_w} R_w^{2/3} S^{1/2} \tag{6}$$

Overall Resistance

We have discussed so far how to separate the frictional resistance of an alluvial channel flow into its component parts, and how each part can be described. This is done chiefly from the view point of sediment transportation, since only the grain resistance is responsible for the bed-load movement. On the other hand, so far as the flow itself is concerned, one often needs a simple equation indicating the overall effect of all systems of roughness, and the parameters of the equation should be easily identifible. There are a few equations belonging to this category, for instance, the Manning's equation

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \tag{7}$$

where n is the overall roughness of the channel. In the older text books of hydraulics, a different value of n is assigned to each type of channel material. This, one now knows, is no longer valid because of the existence of the bar resistance and its dependence on the flow conditions.

Einstein and the writer[30] have proposed a generalized Manning's equation of the following form to replace eq. (7):

$$V = C\sqrt{g}\, R_T^{m+1/2} S^{1/2} K_S^{-m} \tag{8}$$

with

$$R_T = \frac{A_T}{P_b} \tag{9}$$

where A_T is the cross-sectional area and p_b is the channel width. The constant C and the exponent m can be determined graphically, with due considerations of the effects of different components of the frictional resistance. Eq. (8) becomes identical with eq. (7) by taking m = 1/6 and by using the Strickler relation[31]

$$n = K_S^{1/6}/A \tag{10}$$

where

$$C = 1.486A/\sqrt{g} \tag{11}$$

The constant A is generally taken as 29.3 in ft-lb-sec units. The constants c and m in eq. (8) presumably change with the range of R_T/K_S under considerations. A better approximation may be obtained by introducing two sets of c and m, one for the low stages and the other for the high stages. The constants c and m for high stages should not differ materially from eq. (11) and 1/6, respectively, provided that the bank friction is not appreciable.

Another method of describing the overall resistance of an alluvial channel flow has been outlined by Doland and Chow.[32] They retained the use of Manning's equation by modifying eq. (10) to the following

$$n = (K_S^{1/6}/A)/(R_b'/R)^{2/3} \tag{12}$$

where the ratio R_b'/R indicates the effect of sand bars. A graphic solution of R_b'/R has been given in terms of R/K_s and D_{35}/RS, if the bank friction can be ignored.

Sediment Transport Rate

Sediment has been divided into bed-load and suspended-load according to the difference in mode of movement and in the laws which govern its motion; and into bed-material load and wash-load according to the difference in the source of material.

Suspended-Load

Suspended-load is the material moving in suspension in a fluid, being kept up by the upward component of the turbulent current.

Based on the mechanics of turbulent flow of fluids, it has been established that the distribution of the relative concentration of the suspended load is of the following form[9][33]

$$\frac{C_y}{C_a} = \left(\frac{d-y}{y}\,\frac{a}{d-a}\right)^z \tag{13}$$

with

$$z = \frac{V_s}{k\,V_*} \tag{14}$$

where C_y and C_a are the concentrations of a grain size with settling velocity V_s at a distance y and a from the bed, respectively, k is Karman's universal constant, and d is the depth of the flow. Hunt,[34] taking into consideration the space occupied by the sediment particles, introduced a more exact distribution function as follows:

$$\left(\frac{C_y}{1-C_y}\right)\left(\frac{1-C_a}{C_a}\right) = \left\{\left(\frac{1-y/d}{1-a/d}\right)^{1/2}\left(\frac{B_s-(1-a/d)^{1/2}}{B_s-(1-y/d)^{1/2}}\right)\right\}^z \tag{15}$$

where B_s is a constant which was found to be very slightly smaller than unity.

Eqs. (13) and (15) have been shown to describe the suspended sediment distribution sufficiently well, except that the value of the exponent z given by eq. (14) does not agree with z_1, the exponent that fits the measured data.[9][11][35][36] For small z values, the measured exponent z_1 is the same as z. For large z values, the measured values of z_1 are smaller than z, indicating that the distribution of the suspended load is more uniform than that indicated by the theory. Fig. 3 shows the relationship between z and z_1 according to the results of measurements in both rivers and flumes.

Vanoni[9] and Ismail[11] proposed that the sediment transfer coefficient, $\mathcal{E}_s$, is not the same as the momentum transfer coefficient, $\mathcal{E}_m$, but that they are related by the following formula

$$\mathcal{E}_s = \beta\,\mathcal{E}_m \tag{16}$$

where β seems to be a function of the particle size. And

$$z_1 = V_s/(\beta K V_*) = z/\beta \tag{17}$$

which indicates the deviation of z_1 from z. Hunt[34] introduced a different universal constant k_s for suspended sediment distribution such that

$$z_1 = \frac{V_s}{B_s K_s V_*} = \frac{z K}{B_s K_s} \tag{18}$$

No detailed explanations have been given as what basically causes the difference between z and z_1, and what determines the value β or $B_s k_s/k$. On the other hand, Einstein and the writer[18] revised the suspended load theory in the following aspects:

a) The difference in sediment concentration at a vertical distance ℓ apart, where ℓ is the mixing length of the turbulent flow, is given by $\ell\, dC_y/dy$ only for fine particles. For coarse particles, the sediment distribution is highly skewed and the higher derivative terms of the concentration C_y with respect to the vertical distance y have to be included.

b) It is unrealistic to assume that the turbulence eddy maintains its identity until it travels a certain distance ℓ, and only then starts mixing. The model of turbulence eddy is rather to be visualized as if the fluid carried by the eddy continuously mixes with the surrounding fluid on the course of the traverse of the eddy. Only the process of mixing is much more active near the origin of the eddy than at points farther away. In other words, the mixing length is assumed to follow a certain probability distribution instead of possessing a finite value.

c) Since the turbulence is generated at the bed and dissipated toward the water surface, the characteristics of turbulence are not necessarily symmetric. In other words, originated from a certain point, the mixing length and fluctuating velocity of the upward flow might not be the same as those of the downward flow.

The detailed description of the second approximation to the solution of the suspended load theory can not be given here. In one case where the fluctuating velocity and mixing length of the turbulent flow are assumed to follow the normal error law and a certain probability distribution function, respectively, and where the higher derivative terms of the concentration gradient are included, z and z_1 have been shown to be related by the following formula

$$z_1 = \frac{z}{e^{-L^2 z^2/\pi} + \frac{2zL}{\sqrt{2\pi}} \int_0^{\sqrt{\frac{2}{\pi}} Lz} e^{-x^2/2}\, dx} \tag{19}$$

where

$$L = \log_e (1 + BK) \tag{20}$$

and B is a constant. Eq. (19) is plotted in Fig. 3 and the results can be compared with those given by actual measurements. For practical purposes, the difference between z and z_1 perhaps can be ignored since it affects only the coarse particles and the amount of coarse particles in suspension is generally of secondary importance.

In addition to the flow conditions, the settling velocity of the sediment grains is the determining factor in the suspension of material. The general

expression for the settling velocity of sediment particle is as follows:

$$V_s^2 = A \frac{1}{C_D} \frac{\rho_s - \rho_f}{\rho_f} g D \tag{21}$$

where A is a constant and C_D is the coefficient of drag. For large grains which settle with a turbulent wake, C_D is constant. For fall velocity in the viscous range,

$$C_D = \frac{24}{R_e} = \frac{24 \nu}{V_s D} \tag{22}$$

and

$$V_s = \frac{A}{24} \frac{\rho_s - \rho_f}{\rho_f} \frac{1}{\nu} g D^2 \tag{23}$$

where ν is the kinematic viscosity of the fluid. Eq. (23) has been used quite extensively in the size analysis of fine materials by the sedimentation method.

The Constant A in eqs. (21) and (23) depends on a number of factors, for instance, the particle shape, the boundary proximity, the sediment concentration, and the turbulence of the flow. Extensive research has been carried out, not by engineers alone, but by geologists, soil scientists, and chemists as well, to relate A to these factors. To measure the shape factor, geologists used sphericity[37] which is defined as the ratio of the surface area of the sphere having the same volume as the particle to the actual surface area of the particle. For practical purposes, the sphericity of a particle can be approximated by combinations in various forms of the maximum, intermediate, and minimum mutually perpendicular axes. Some of the recent works in this phase of the problem include those by Krumbein,[38] McNown, Malaika and Pramanik,[39] and Albertson.[40] The effect of the boundary proximity on the settling velocity has been summarized, together with other effects, by McNown.[41] McNown and Lin[42] also showed that for a 0.1 mm. sand at a concentration of 30 gm/lit. the decrease in fall velocity is as much as 20 %. The effect of very high sediment concentration has been studied by Steinour,[43] and Wilson.[44] Sediment mixtures settle collectively as a unit at very high concentrations despite of the spread in particle sizes. Hawksley[45] showed that there is a continuity from the effect of sediment concentration on the settling of suspensions to the flow through porous media, the latter being the upper limit of the former. When sediment settles in a turbulent flow, not only the particle itself, but also the turbulent wake behind the particle, is subjected to the acceleration and deceleration of the flow. This introduces the virtual mass problem and has been studied by Iversen and Balent.[46] A recent investigation by the Missouri River Division, Corps of Engineers indicated that for the turbulence generally existing in natural streams, its effect on the settling velocity is of minor importance.

In spite of the fact that the problem in relating the properties of the sediment and fluid to the fall velocity of the particles has received more attention than any other phase of sediment problem, much still remains to be done.

Most of the investigations of which only a small part is cited here originate from the view point of sediment size analysis and is, therefore, confined to fine particles in the Stokes' range. Much more important to river engineers is the behavior of coarse particles, the transport rate of which is a governing factor on channel stability. How the various factors affect the settling velocity of these coarse particles deserves special considerations.

The suspended-load theory breaks down near the bed where the particle size is of the same order of magnitude as the size of the eddies around the particle. The particle merely settles down between eddies without being lifted up again by the surrounding fluid masses. The thickness of the bed layer in which suspension becomes impossible has been found to be about two grain diameters. Within this layer, the particles are pushed forward by the flow. Their weight is no longer supported by the flow, but by the bed itself, and the motion of the particles takes the form of rolling, sliding, and occasionally, jumping. This part of load is generally called the bed-load.

Bed-Load

The bed-load movement has received much attention by many investigators, and almost every investigator has proposed a formula of his own, relating the bed-load transport rate to the flow conditions. A historical development of these bed-load formulas has been outlined by Vanoni,[47][48] and only the most important ones will be given here in their final forms. Attempts will be made to point out how one can achieve a certain degree of unification out of the various formulas.

1. Tractive Force Theory—The most common conception of the mechanics of bed-load movement is that a dragging force exists on the stream bed due to the flowing water. This force, termed the tractive force, is the entraining force exerted at the base of a prism of water of unit area of the bed and of height equal to the water depth sliding, under the influence of gravity, down an inclined plane having a given slope. When the tractive force is sufficient to overcome the resistance of the particles, sediment begins to move as bed-load. The force which creates the commencement of movement of the bed material is called the critical tractive force.

The idea of tractive force was first introduced by DuBoys[1] in the last century. Since then numerous flume studies have been conducted to obtain a relationship between the various factors influencing the beginning of bed-load movement. Some of the results were summarized by Chang.[49] A recent analysis by Carter[50] indicated that most of the dimensionally correct critical tractive force formulas can be reduced to the form

$$\frac{T_o}{(\gamma_s - \gamma_f)D} = A \qquad (24)$$

where T_o is the critical tractive force, γ_s and γ_f the sp. wt. of sediment and fluid, respectively, D the grain size and A is a constant. Shields[25] showed that A is actually a function of D/δ, where δ is the thickness of the laminar sublayer. A becomes a constant only when there is fully developed turbulent flow at the bed.

It has been shown[51] that the "critical" conditions for the commencement of sediment movement actually correspond to a particular value of flow intensity ψ, the parameter introduced by Eintein[3] in his bed-load function.

Fig. 6, after Lane[52] and Carter,[50] shows a plot of the various critical tractive force formulas. Eq. (24) gives a straight line of 1 to 1 slope in Fig. 4. The value Ψ, corresponding to the "critical" conditions, varies considerably for results of different investigators and is given in Fig. 4. A closer survey of these formulas reveals that the so-called "critical" value in some cases is a practical value, indicating zero transport. In some other cases, it is a constant of the transportation law. These two are so different in nature that one should not be astonished by the apparent spread of the "critical" value.

Based on the idea of critical tractive force, the sediment transportation should be a function of the difference between the tractive force and the critical tractive force of the material in the bed. Thus, DuBoys proposed his classical bed-load formula as follows:

$$q_B = C_B \tau (\tau - \tau_0) \tag{25}$$

where q_B is the transport rate measured in weight per unit width and time, T the tractive force, and C_B is a sediment parameter. DuBoys based his analysis on the oversimplified picture of sliding layers of bed material kept in motion by the tractive force of the moving fluid. His reasoning was improved by O'Brien and Rindlaub[53] to give the bed-load formula of the type

$$q_B = K(\tau - \tau_0)^m \tag{26}$$

where K and m are constants. Eq. (26) has been used by the US Waterway Experiment Station.[54] The exponent m varies from 1.4 to 1.8 depending on the material tested.

There are numerous other formulas derived directly or indirectly from DuBoys tractive force theory which are different in form from eqs. (25) and (26). Some of the formulas have been tabulated by Vanoni[47] and Brown.[55] Johnson[56] has shown that they all fit the data of flume experiments equally well.

Kalinske,[57] using the idea of critical tractive force and applying some of the results of modern turbulence research, developed a theory for bed-load transport as follows:

$$q_B / (v_* \gamma_s D) = f_2(\tau_0/\tau) \tag{27}$$

where the function f_2, which involves the characteristics of the fluid turbulence, is analytically given. Mostafa[58] has shown that eq. (27) is similar to the Einstein bed-load function, with the difference lying in the definition of a velocity scale.

2. Einstein Bed-Load Function—It was first concluded on the basis of special experiments that a given particle size moves in a series of steps of a constant average length, and that the particles are periodically deposited in the bed after performing such a step. The number of particles deposited per unit of time in the unit of bed area may be expressed in terms of the rate of transport and the size and weight of the particles. The rate at which these sediment particles are eroded from the bed is proportional to the number of particles in the surface of the bed area and to the probability that such a particle is eroded during the unit time. This probability is then expressed as a function of the ratio of dynamic lift on the particle to the weight of the particle under water. The equilibrium rate of bed load transportation is then

obtained by equating the number of particles deposited on and eroded from the unit bed area per unit time. This leads to a functional relationship between the flow intensity, ψ, and the intensity of bed-load transport, ϕ,

$$\phi = f_3(\psi) \tag{28}$$

Eq. (28) was defined empirically in 1942,[59] and was replaced in 1950[3] by an analytical function as follows:

$$1 - \frac{1}{\sqrt{\pi}} \int_{-B_* \psi - 1/\eta_0}^{B_* \psi - 1/\eta_0} e^{-t^2} dt = \frac{A_* \phi}{1 + A_* \phi} \tag{29}^2$$

where

$$\phi = \frac{q_B}{\rho_s g} \left(\frac{\rho_f}{\rho_s - \rho_f}\right)^{1/2} \left(\frac{1}{g D^3}\right)^{1/2} \tag{30}$$

$$\psi = \frac{\rho_s - \rho_f}{\rho_f} \frac{D}{R_b' S} \tag{31}$$

and A_*, B_* and η_0 are universal constants, with[2]

$$A_* = 1/0.023 = 43.5, \qquad B_* = 1/7 = 0.143, \qquad 1/\eta_0 = 2$$

3. Meyer-Peter Formula--The Meyer-Peter formula was developed at the Zurich Hydraulic Laboratory and has been used quite extensively in Europe. It was first published in 1934[60] as follows:

$$\frac{q_b^{2/3} S}{D} = a + b \frac{q_B^{2/3}}{D} \tag{32}$$

where

q_b = V R_b, V is the average velocity of the flow and R_b is the hydraulic radius of the bed

a, b = constants

Eq. (32) is derived empirically, using Frouds law of similarity as the guide as it can be seen that the quotient $q_b^{2/3}S/D$ is independent of the scale since both the numerator and denominator vary with the scale in a linear manner. In 1948, Meyer-Peter and Muller[7] modified eq. (32) in two respects, namely, the inclusion of the effect of sediment density and the separation of the bar resistance from the total bed resistance, to give a new formula of the following form

$$\gamma_f \frac{q_b}{q} \left(\frac{K_b}{K_b'}\right)^{3/2} d S = A(\gamma_s - \gamma_f) D + B \left(\frac{\gamma_f}{g}\right)^{1/3} \left(\frac{\rho_s - \rho_f}{\rho_s}\right)^{2/3} q_B^{2/3} \tag{33}$$

2. Note that eq. (29) and the numerical values of the universal constants as given in Ref. (3) involve certain typographical errors.

where

$q = Vd$
d = average depth of flow
$K_b = V/(R_b^{2/3}S^{1/2})$
$K_b' = V/(R_b^{2/3}S_b'^{1/2})$
A, B = constants
q_B = sediment transport rate per unit width per unit time measured by dry weight

The division of the bed resistance into its component parts is accomplished by holding R_b constant and dividing the energy slope, S, into two parts—the energy slope with respect to the grain resistance, S_b', and the energy slope with respect to the bar resistance, S_b''.

The writer[51] has shown that eq. (33) can be modified to give the following relationship between ϕ and ψ—the same parameters introduced by Einstein in his bed-load function:

$$\phi = \left(\frac{4}{\psi} - 0.188\right)^{3/2} \tag{34}$$

Eq. (34), together with eq. (29), is plotted in Fig. 5. It shows excellent agreement between the Einstein bed-load function and the Meyer-Peter formula up to a ϕ value of 10. With larger ϕ values, the Meyer-Peter formula gives a larger transport rate than that indicated by the Einstein bed-load function.

The experimental results plotted in Fig. 5 were obtained under the following range of conditions:

Slopes from 0.15 to 20%
Particle sizes from 0.8 to 30 mm.
Water depth from 0.03 to 4 ft.
Discharges from 0.0215 to 21.5 cfs/ft.
Sp. Gr. of sediment from 1.05 to 4.2.
Particle shape from near spherical to rectangular

The fact that the experimental points, derived from such a wide range of variables, fall exactly on the curves clearly demonstrates the merit of both methods.

Eq. (33) also can be reduced to the form[51]

$$q_{gB} = K_1 (\tau - \tau_c)^{3/2} \tag{35}$$

which is identical with eq. (26), one of the formulas derived from DuBoys tractive force theory, with m = 3/2.

Meyer-Peter formula also has been shown[51] to be similar to the Bagnold equation[20] for sediment transport in air, taking into consideration the large difference in density between air and water.

One now sees that the various bed-load transport formulas as proposed by different investigators are actually of compatible form and consist of the same parameters.

4. Sediment Mixtures—All the theories we have discussed so far were derived originally from experiments with uniform sediment. For any given sediment mixture there is some grain size which, using these theories, will give the same rate of movement as the mixture does. While Einstein[61] proposed to use D_{35} as the effective diameter, Meyer-Peter and Muller[7] suggested the use of a sediment size of the following value

$$D_m = \Sigma D \Delta P/100 \tag{36}$$

where Δ p is the percentage by weight of the bed material with grain size D. Kalinske[57] recommended that the medium diameter be used. At large transport rates it becomes immaterial which one of these is regarded as the effective size.[51]

The nature of the sediment problem is often such that one needs to know the composition of the bed-load as well as the total transport rate. Basically one may still apply the normal bed-load equations to the different components of a bed, with due regards to the mutual interference effects between bed particles of different sizes. For instance, fine particles may hide behind large protruding particles at the bed surface and, in consequence, are not subject to the same velocity as in the case where the whole bed is composed of material of only that size. This effect differs considerably, depending on whether the bed material is homogeneous, or is locally sorted.[62] Einstein and the writer took into consideration the "interference-effect" of sediment mixtures by introducing a set of correction factors. In applying Einstein's bed-load function to the components of a mixture, the flow intensity and the intensity of bed-load transport should be modified to assume the following form:

$$\Phi_* = \Phi i_B/i_b \tag{37}$$

and

$$\Psi_* = \frac{\xi \gamma (\beta/\beta_x)^2}{\theta} \Psi \tag{38}$$

where

i_B = fraction of bed-load in a given grain size range
i_b = fraction of bed material in a given grain size range

and ξ, Y, $(\beta/\beta_x)^2$, and θ are correction factors introduced for non-uniform materials.[3][62] These correction factors, although the only working tool available at the present, should by no means be considered as the best solution. In fact it is anticipated that they may be considerably simplified if some phases of the sediment transport problem, for instance, the effect of the heavy sediment concentration near the bed on the flow and sediment motion, becomes known.

Evaluation of Total Load

So far sediment motion has been described separately as suspended-load and bed-load, because of the difference in laws which govern its motion. This does not imply that the two modes of transportation are completely unrelated. Indeed sediment motion must be continuous from the bed up to the water surface. Not only the bed-load, but the suspension as well, must relate to the composition of the stream bed.

Currently there exist two methods in linking a point in the flow to a point on the bed. Lane and Kalinske[63] argued that sediment can only be kept in suspension when it is available on the bed and when the vertical fluctuating velocity of the turbulent flow is larger than the settling velocity of the particle. The rate at which particles of a given size are picked up and placed in suspension is, therefore, proportional to (a) the relative amount of these particles in the bed, (b) the magnitude of the vertical velocity presented and capable of picking them up, and (c) the relative amount of time during which velocities capable of picking up particles of that size exist. This approach

thus links a point in the flow directly to a point on the bed, and implies that suspended-load theory can be extended down to the bed. On the other hand, Einstein[3] has argued that sediment cannot be kept in suspension close to the bed where the size of the eddies becomes comparable to the size of the particles. The layer close to the bed within which suspension becomes impossible has been termed the "bed-layer," and it has been shown how the flow together with the sediment composition of the bed determines the transport of sediment as bed-load in the bed layer. The relationship was found to be governed by the exchange of sediment particles between the bed layer and the bed. In a similar manner, one can also establish the relationship governing the concentration at the lower edge of the suspension (i.e., at the upper boundary of the bed layer) by setting up an expression for the exchange of sediment particles between the suspension and the bed layer. Taking this concentration as the reference concentration C_a where a is the thickness of the bed layer, one can use eq. (13) to determine the distribution of suspended load along the vertical. In this way a point in the flow is linked directly to a point in the bed layer, and thereby indirectly connected to a point on the bed. The transport rate of suspended-load is obtained by integrating the product of local velocity and sediment concentration from the bed layer up to the water surface. Knowing the rates of transport of suspended-load and bed-load, the total sediment discharge moving across the section is merely the sum of the two.

Wash Load[64]

Basically, the sediment supply is the primary factor determining the behavior of the stream. The channel will be aggraded or degraded depending on whether the sediment inflow into the reach is larger or smaller than the capacity of the channel to transport the material. In the past sediment has been furnished, and usually it still is being furnished, by the watershed at a rate that is higher than the capacity load. Most streams are continuously subjected to a very slow process of aggradation.

Recent studies on the behavior of aggrading channel[62] seem to indicate that (a) coarse particles tend to accumulate in the entire bed, and a very large percentage of the material in the bed maintains a small percentage of the same material in motion; and (b) fine particles tend to remain at the bed surface, and its presence in the bulk of bed deposits is insignificant. Although fine particles constitute only a small part of the bed, they represent most of the sediment in motion.

In the course of centuries the stream channel has been continuously built up by sediment deposits of which the coarse particles contribute the great majority. If for any reason the sediment inflow temporarily changes, the local deposition or scour will not materially affect the amount of coarse particles available at the bed surface. Any additional particles deposited on the bed surface will be stored in the bed, and any particles removed from the bed surface will be quickly replaced from the bed beneath. Since the bed composition essentially remains constant and if the change in sediment supply is only of short duration, the sediment transport rate is determined solely by the flow conditions. By the same reasoning, although it must have been eroded or otherwise have been made available in the watershed, the immediate source of the coarse material comes from the upstream reach of the channel. For this type of material one uses the special name "bed-material load."

On the other hand, the fine particles accumulate only in the surface layer

of the bed in any significant amount. Even if this small percentage of fine particles in the bed permits the prediction of the large rate of such particles moving with the flow at that instant,[65] it is not sufficient to make up for a lack of supply of such particles from upstream, nor does the bed give room for the deposit of a surplus in the upstream supply. The bed composition will thus change with the supply; and the transport, representing the major part of the supply, will not be affected by temporary bed storage, but will fluctuate with the supply. This type of the material comes all the way from the upstream watershed. On its route to a given reach, a very small part of it may settle out or it may pick up some from the channel bed, in accordance with the local conditions. But essentially it maintains its identity, and its transport rate depends, to a larger degree, upon how much it is produced at its points of origin in the upstream watershed. In contrast with the bed-material load, this type of the material is called "wash-load."

The differentiation between the "bed-material load" and the "wash-load" has its practical significance. Consider, for instance, that the regime of the channel is upset by a permanent change in sediment supply. This will lead to certain changes in channel characteristics until the capacity of the channel to transport the sediment equals the sediment supply, and only then will the channel attain a new equilibrium state. If the change in supply involves only the wash-load material, the channel takes only a slight modification of its composition at the bed surface. If, however, the change in sediment supply involves the bed-material load, then the channel must undergo a significant modification of its characteristics, involving substantial deposition or scour of the channel bed. The transport rate of the bed-material load, therefore, is the governing factor on channel stability. On the other hand, wash-load, which has a minor effect on the channel stability, contributes the bulk of the deposition in lakes and reservoirs.

The rate of transport of wash-load depends essentially on the different factors determining the availability of the material from the watershed. Anderson,[66] in analyzing 29 watersheds of Western Oregon, has associated quantitatively the variation in the suspended sediment discharge (in which presumably the wash-load constitutes the major part) with differences in stream-flow, topography, soils, land use, and bank conditions of the watershed. This type of analysis should definitely be extended to take advantage of the great amount of reservoir and basin survey data accumulated in the last two decades by various federal agencies,[67] such that a general relationship between the amount of wash-load and the characteristics of the watershed may finally be established.

Other Phases of Sediment Research

Owing to the diversity of interests of research workers and to the complexity of the sediment problem itself, the horizon of sediment research has broadened greatly in recent years. Because of the limitation of space, only those problems which are related to sediment transportation in open channels will be discussed here in a very brief manner.

Field Sampling

All the sediment transportation equations were developed from flume experiments in the laboratory, and it is very important to determine their applicability to natural streams by making field measurements. Field sampling today consists mostly of suspended sediment measurement by either depth-

integrating or point-integrating samplers. The depth-integrating samplers cannot sample down to the bed, and, therefore, trap only a part of the total sediment load moving across the vertical. Methods[68][69] have been developed to correct this inherent error of sampling. For small streams, there is also a method[70] of causing, by natural or artifically means, all the material to go into suspension such that the suspended-load measurement will yield total transport rate. Comparisons of field data with existing theories[71] seem to favor the Einstein bed-load function and the Meyer-Peter formula. It has been pointed out[72] that almost all of the flume studies are confined to relatively small depths and steep slopes. The ratio of flow depth to the bed material grain size is comparatively small. These conditions correspond to a stream with a steep slope, a shallow water depth, and a large sediment size. The constants and functions of the sediment transport formulas as determined from the flume studies may have to be modified in applying the formulas to deep rivers with a small bed material grain size and flat slopes. And it is in this range of conditions that the field measurements are most urgently needed.

The current trend in determining the total sediment transport rate of a river reach, in connection with the basin planning, is to combine the analytical method with the suspended sediment sampling program. The transport rate of coarse particles is calculated according to the existing theories, and the measured results are used to determine the transport rate of fine particles. This is a logical step since (a) coarse particles move essentially close to the bed and cannot be sampled by suspended-load samplers, and (b) fine particles behave like wash-load and their transport rate cannot be predicted by analytical methods.

Channel Cross Section

One of the most basic questions confronting a river engineer is the following: given the bed material, what should be the width, depth, and slope of the channel to accommodate a given discharge and a given sediment inflow? In designing canal systems with rigid boundaries and without sediment motion, two of these three variables must be known. For alluvial channels, the condition to transport sediment at a certain rate fixes another variable, according to the theories as depicted in the preceding two sections. Fig. 6, for example, shows how the depth and slope can be determined from the given unit discharge and total sediment transport rate, assuming a representative bed material size of 0.25 mm.[73] In most cases a change of sediment load, say by 50%, does not materially affect the slope and the depth. So far as the design of alluvial channels is concerned, considerable tolerance can be allowed in the determination of sediment transport rate. This clearly demonstrates that although efforts should be made to improve the existing sediment transport formulas, any hasty rejection of these formulas because they in cases fail to give close agreement with the measured data is highly unwarranted. Graphs like Fig. 6 also can be used as valuable aid in gaining insight into the change of river morphology by engineering measures.

On the other hand, there is still one variable which remains undetermined. What, for instance, should be the width of a natural stream channel? As early as 1919 Lindley[74] stated that the "dimensions, width, depth, and gradient of a (regime) channel to carry a given supply loaded with a given silt charge were all fixed by nature." A channel in regime means a channel at equilibrium. Since then, based on observations on the behavior of irrigation canals, engineers in India have succeeded in developing a set of rules

which control the cross section and gradient of regime channels. For instance, Lacey claimed that the wetted perimeter, p, is related to the discharge, Q, by the following formula,

$$P = 2.67Q^{1/2} \tag{39}$$

Investigations by Leopold and Maddock[75] indicated that for natural streams the constant in equation (39) should be, among others, a function of the bed material.

The mass of data and experience gathered by engineers in India is so large that even a brief outline here will overburden a paper of this nature. Reference is made to Lacey's original works[76][77] and a summary of the regime theory by Blench.[78] A comparison between the Indian regime theory and the Einstein bed-load function is currently being undertaken by the writer. The results of preliminary analysis seem to indicate that within the range of conditions prevailing in India, the Indian regime theory, by properly selecting the various "silt factors," gives practically the same results as those given by the Einstein bed-load function. Outside of the range of conditions in India, the use of regime theory is difficult since the "silt factors" then depend not only on the characteristics of sediment, but also on the hydraulic characteristics of the channel.

Leopold and Maddock have developed an empirical quantitative relation among width, depth, velocity, discharge, and suspended sediment load from data on natural streams in the United States. The inclusion of sediment load in this type of empirical formulas is significant and interesting. Application has been made of the Leopold and Maddock formula in the actual design of stream channel.[79]

In connection with the above problem is the design of stable channels. The regime channel, by definition, is stable. Progress also has been made by U.S. Bureau of Reclamation, under the direction of Lane[52][80][81] in clarifying the general principles of stable channel design and in working out methods of designing unlined earth canals to insure freedom from scour.

Density Current

A density current is a gravity flow of a liquid or gas through, under, or over a fluid of approximately equal density.[82] The density difference is due to dissolved solids, to temperature differences, and to suspended sediment. Geologists and oceanographers have used a term "turbidity current" to identify a particular type of density current which contains suspended sediment and flows along the floor of a still body of clear water.

The most intricate feature of a turbidity current is its ability to maintain its identity. The stability of a turbidity current involves two aspects: that is, (a) the stability at the interface of the turbidity current and the flow above, and (b) the stability of the current itself. The first phase of the stability problem has been studied by a number of research workers, including Keulegan,[83][84][85] Ippen and Harleman,[86] and Long,[87] among many others. On the other hand, it still remains a mystery today how underflows with Reynolds numbers as high as 100,000 or more are not mixed with the flow above. Certain mechanisms must exist which prevent the turbulence created along the bed from penetrating the interface. Einstein[88] suggested that at high sediment concentrations and especially when sediment is flocculated, the underflow is no longer viscous but becomes plastic. And the plasticity of the flow appears to be highly responsible for the existence of

non-mixing underflows at high Reynolds numbers. Recent analysis of the structure of the sediment suspended in Lake Mead[89] indicated that the sediment does travel in a flocculated state.

Since the passage of turbid water through the tunnel outlets of Hoover Dam was first reported in 1938 by Grover and Howard,[90] considerable interest has been stimulated on the density current as a possible means of relieving the rate of reservoir silting. A follow-up survey of sedimentation in Lake Mead revealed that at times substantial portions of the inflow into Lake Mead travelled along the bottom of the reservoir, following the old river bed, and resulted in a deposit over 100 ft. thick.[91][92] However, to rely on the density current as an effective means in releasing sediment from reservoirs requires practical considerations. A density current is possible only when there is high sediment concentration in the inflow. To release the sediment carried by a density current, one must open the outlet by the time the density current reaches the dam. Once the sediment in the density current is settled out of suspension, it has such a high resistance against scour that further flushing will only cut a deep gorge near the outlet and will have little effect on the deposits elsewhere. The design and maintenance of large outlets located at low levels in a dam is not a simple matter, and the high erosive action resulting from flow with large sediment concentration moving at relatively high velocity must be considered. Needless to say a considerable amount of water will be lost from the impounding reservoir in passing a density current through the reservoir.

Perhaps some thought should be given to another phase of the turbidity current, namely, the low unit weight of the reservoir deposits brought in either by density current or by other means. Consider Lake Mead, for instance, the overall unit weight of the deposits is 64.8 pounds per cu. ft. However, the deposits of fine-textured sediment have extremely high porosities and low specific weights.[89] One of the cores showed that in the first 15 ft. of deposits, the sp. wt. varied from only 20 to 40 pounds per cu. ft. If some means can be devised to release the water trapped in the deposits and thereby increase the unit weight, the space of the reservoir lost to the sediment deposits will be very much reduced. In the field of soil mechanics, there are means to compact the soil[93] for better foundations. These methods, as they stand now, cannot be applied to compacting the reservoir deposits because of the high costs involved and relatively low efficiency. However, with the problem of reservoir silting becoming increasingly serious and with proper dam sites becoming more and more scarce, these methods, or others, should be studied and improved and may eventually prove feasible in attempting to increase reservoir life.

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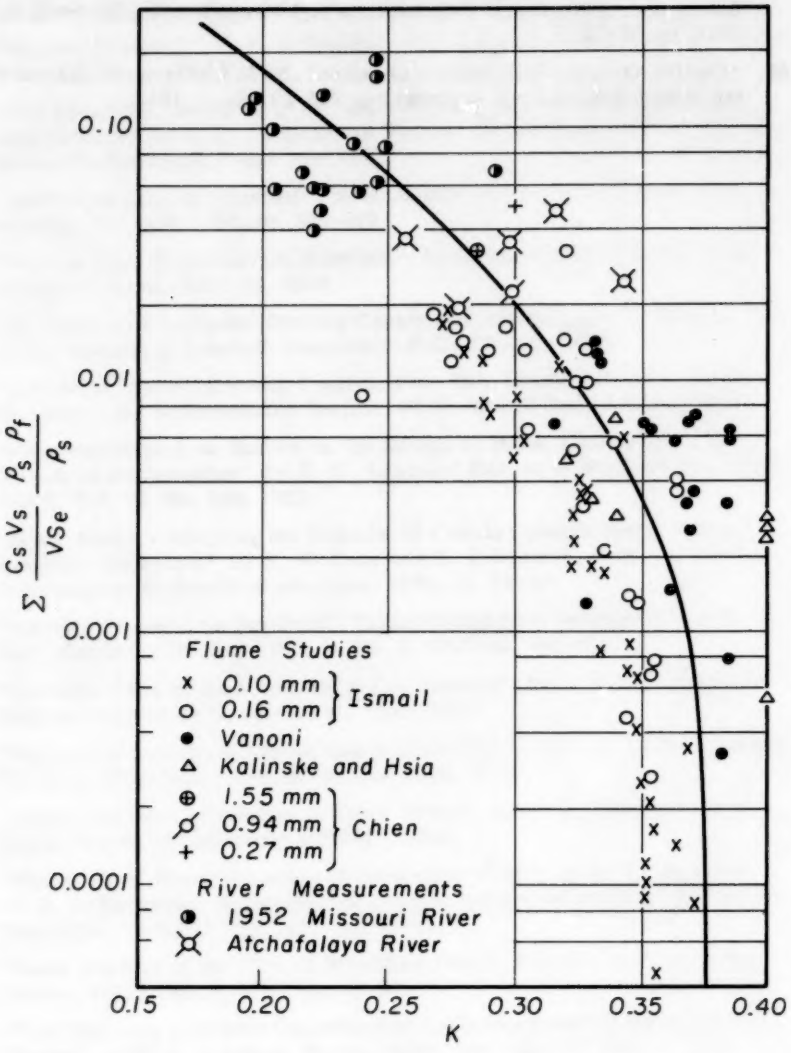


Fig. 1 Effect of Suspended Sediment on Velocity Distribution.

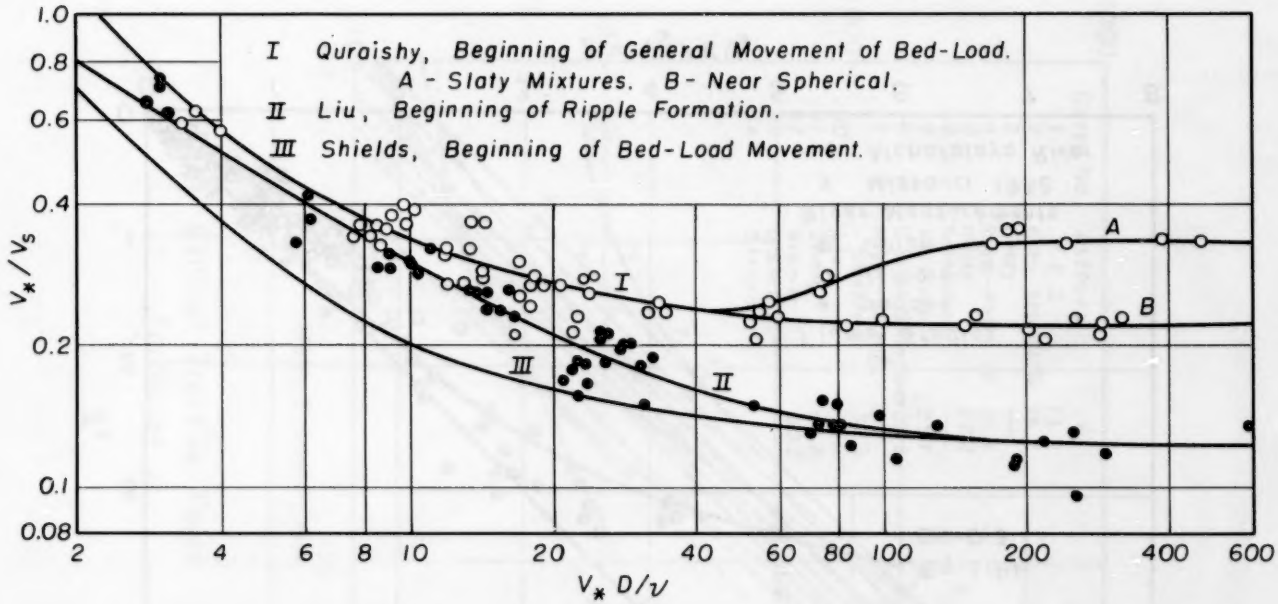


Fig. 2 Beginning of Bed-Load Movement and Ripple Formation.

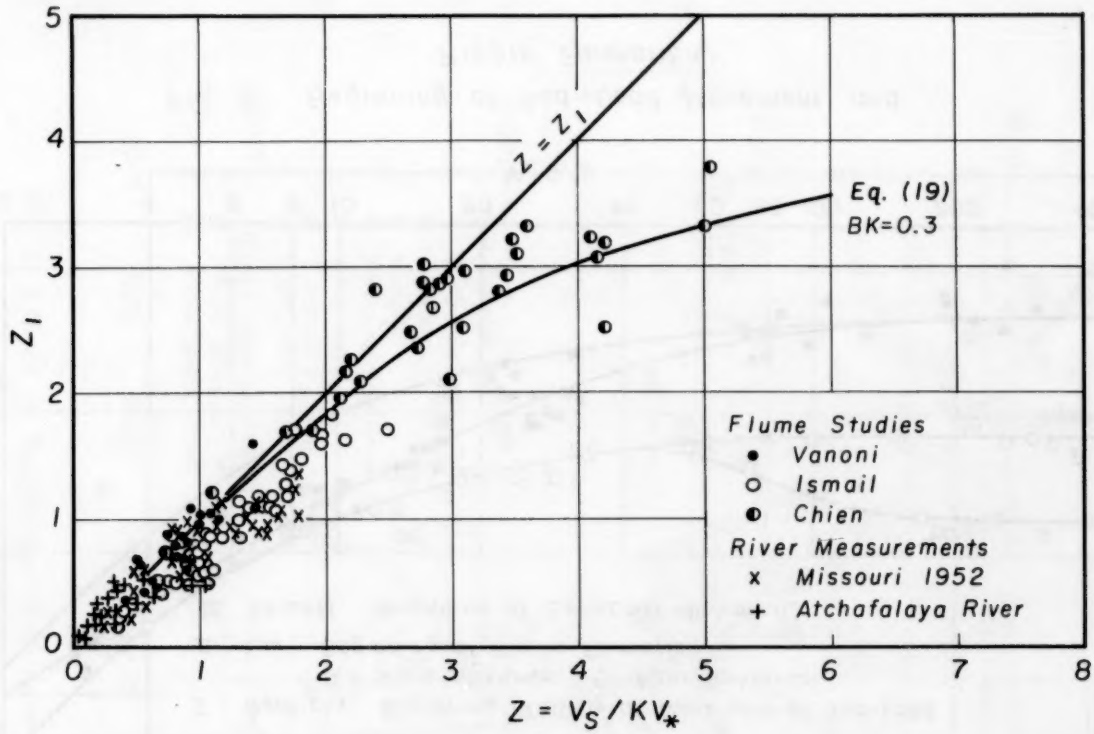


Fig. 3 Relationship Between Z and Z_1

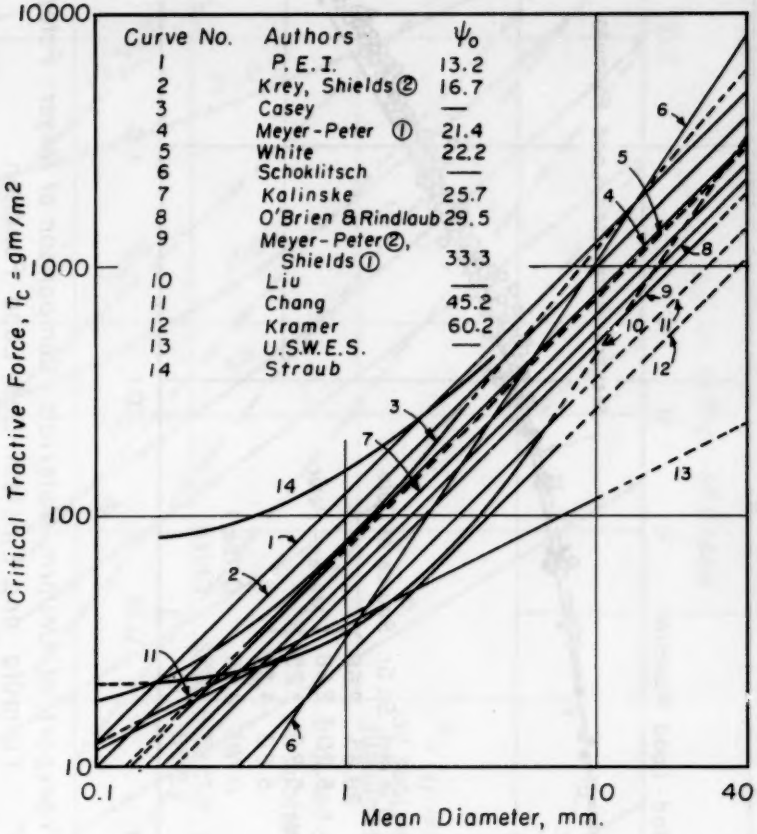


Fig. 4 Critical Tractive Force Formulae

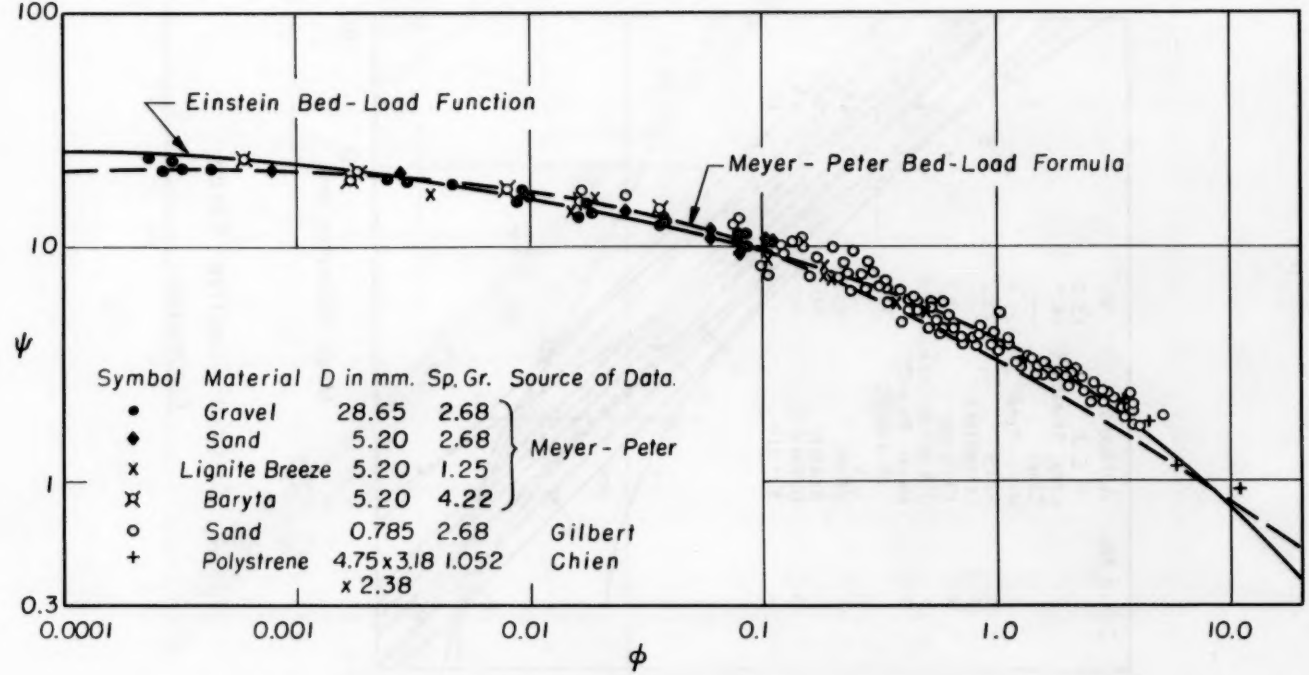
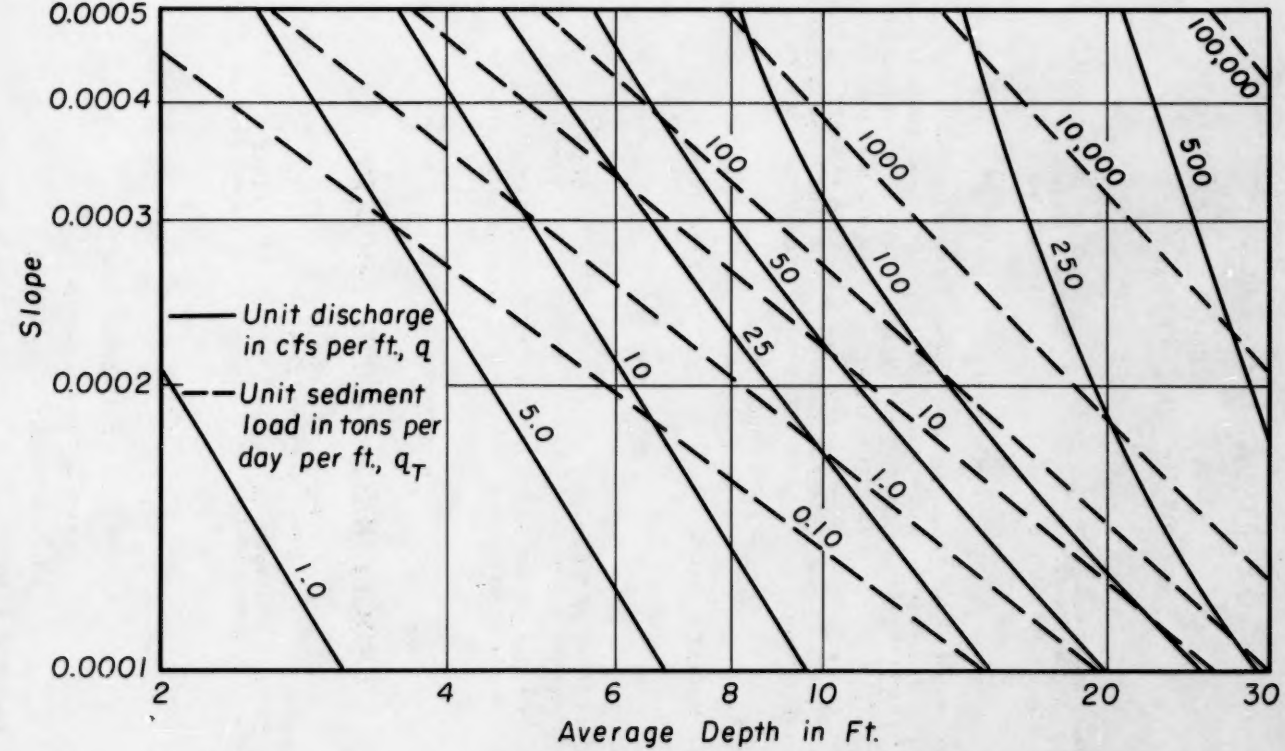


Fig. 5 Transport of Uniform Materials - Comparison of Meyer-Peter Formula and Einstein Bed-Load Function.



565-33

Fig. 6 Design of Alluvial Channels to Carry the Given Discharge and Sediment Load Fine Sand Bed (D = 0.25 mm.) Bank Friction Neglected.

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